Rhine Demolition
3rd and Columbia Demo
823 3rd Ave. Building

Seattle, WA

Structural Calculations

CALCULATIONS INCLUDED:

These calculations cover the rubble support of existing basement wall and demolition sequencing of diaphragm for the 823 3rd Ave building. Demolition of the building is in advance of soldier pile shoring installation around the site.

The existing 823 building has one below grade level. When the building is demolished, the basement walls will be supported by pushing the concrete demolition debris into a berm against the walls to provide resistance against the forces from the soil.

Hart Crowser has provided resistance values for the rubble berm for both sliding and passive failure modes in the geotechnical report included in these calculations. KPFF has performed calculations showing that the rubble berms are adequate to support the basement walls using these values, and has included an additional factor of safety of 1.25 above what is recommended by Hart Crowser due to the importance of 3rd Ave.

KPFF has evaluated the demolition sequencing and found that when most of the west portion of the buiding is demolished, the remaining portion of the diaphragm along the east face of the building can span between the perimeter walls, acting as a beam to support the soil loads on the east side of the building.

kpff

1601 5th Avenue, Suite 1600
Seattle, WA 98101 206.622.5822

GREGORY L. VARNEY
STATE OF WASHINGTON
25842
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Greg Varney, S.E.

Digitally signed by Greg Varney, S.E.
Reason: I am approving this document
Contact Info: greg.varney@kpff.com
Date: 2021.03.10 14:17:00-08'00'

KPFF Project No. 10042100015.10
3/10/2021

kpff
1601 5th Avenue, Suite 1600
Seattle, WA 98101 [206] 622-5822

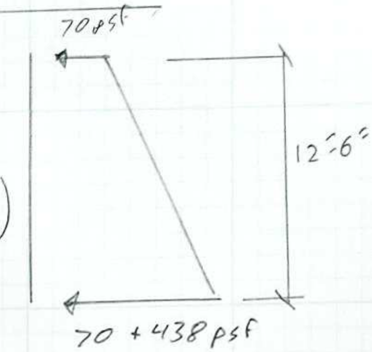
project 3rd & Columbia		by SLY	sheet no.
location		date 2-24-21	job no.
client			
Design of Rubble Berm			

North wall 12'-6" Tall section, Also 3rd Ave North Side

height of retained soil = 12.5 Ft

Total load from soil = $70psf \cdot 12.5ft + 438psf \cdot 12.5ft \cdot \frac{1}{2}$

= 0.88 k + 2.74 k = 3.62 k



weight of material used to resist sliding (NOTE: 1.625 factor is F.S. per HC x an add'l. 1.25 F.S.)

$3.62k \cdot 1.625 = 0.3 \cdot W \Rightarrow W = 19.6 k$

Try a berm that starts at the top of wall & slopes down @ 1.5H:1V

$A = 12.5ft \cdot (12.5ft \cdot 1.5) \cdot \frac{1}{2} = 117\ ft^2/ft$

if density = 110 pcf wt = $117ft^2 \cdot 0.11kcf$ = 12.9k n.g.

if density = 125 pcf wt = $117ft^2 \cdot 0.125\ kcf$ = 14.6k n.g.

with added 6' wide bench

110 pcf wt = $12.9 + 12.5 \cdot 6 \cdot .11 = 12.9 + 8.25$ = 21.2 klf ok

125 pcf wt = $14.6 + 12.5 \cdot 6 \cdot .125 = 14.6 + 9.38$ = 24.0 klf ok

Passive resistance

$K_p = 4.1$ (interpolated from HC report)

Passive resistance = $(12.5 \cdot 4.1 \cdot 0.11kcf) \cdot 12.5ft \cdot \frac{1}{2} = 35.2k$

Factor of Safety = 2.0 (from HC) $\cdot$ 1.25 = 2.5.

Passive resistance / F.S. = 35.2k / 2.5 = 14.1k > 3.62k ok

kpff

1601 5th Avenue, Suite 1600
Seattle, WA 98101 [206] 622-5822

project 3rd & Columbia | by SLN | sheet no.

location | date 2-24-21 | job no.

client

Design of Rubble Berm

North wall 8 ft of retained soil

total load from soil $= 70psf \cdot 8ft + 280psf \cdot 8ft \cdot \frac{1}{2} = 0.56k + 1.12k = 1.68k$

Weight of materials needed to resist sliding

$1.68k \cdot 1.625 = 0.3 \cdot W \Rightarrow W = 9.1\ K$

if berm is 1.5H:1.0V from top of grade

$A = 8 \cdot (8 \cdot 1.5) \cdot \frac{1}{2} = 48\ ft^2/ft$

110 pcf $wt = 48ft^2 \cdot 0.11\ kcf = 5.28\ klf$ $\boxed{\text{n.g.}}$

125 pcf $wt = 48ft^2 \cdot 0.125\ kcf = 6\ klf$ $\boxed{\text{n.g.}}$

w/ added 6 ft bench

110 pcf $wt = 5.28\ klf + 8 \cdot 6 \cdot .11 = 5.28 + 5.28 = 10.6\ k$ OK

Passive resistance

$K_p = 5.4$ (interpolated from HC report)

passive resistance $= (8 \cdot 5.4 \cdot 0.11kcf) \cdot 8 \cdot \frac{1}{2} = 19.0k$

Passive resistance / F.S. $= 19.0k / 2.5 = 7.6\ k > 1.68k$ OK

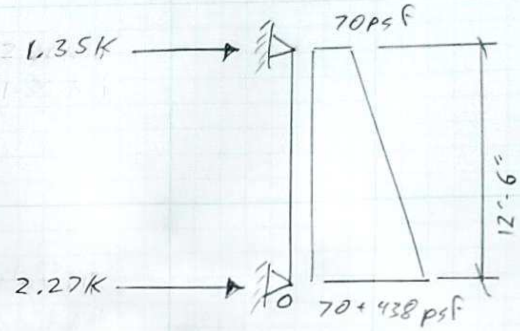
kpff
1601 5th Avenue, Suite 1600
Seattle, WA 98101 [206] 622-5822

project: 3rd & Columbia | by: SLN | sheet no.:
location: | date: 3-8-21 | job no.:
client:

Diaphragm Span in Temp Condition

823 building

Evaluate depth of diaphragm needed to support loads from 3rd ave.



$\Sigma M_O = 0.875k \cdot \frac{12.5}{2} + 2.748k \cdot \frac{12.5}{3} = 5.47k\cdot ft + 11.45k\cdot ft$
$= 16.92\ k\cdot ft$

$R_T = 16.92 k\cdot ft / 12.5 = 1.35\ k/ft$

$R_B = 0.875 + 2.748 - 1.35 = 2.27$

Reaction @ T / wall = 1.35 k·ft

Building is 117 ft wide

$M = 1.35 k/ft \cdot (117\ ft)^2/8 = 2310\ k\cdot ft$

$V = 1.35\ k/ft \cdot 117\ ft\ /2 = 79.0\ K$

Per structural notes $f_y = 60$ ksi reinforcing $f'_c = 3$ ksi

$M_u = 1.6 \cdot 1.25 \cdot 2310 k\cdot ft = 4620\ k\cdot ft$ $V_u = 1.6 \cdot 1.25 \cdot 79K = 158K$

Courtyard slab Thickness = 4½" (typ joist detail on S7) Reinf = #5 @ 14" EW ctr. of slab (Typ joist detail on S7)

Joists are @ 3 ft o.c.
Check slab usage of steel
assume 100 psf dl 25psf LL $w_u = 1.2 \cdot 100 + 1.6 \cdot 25 = 160$ psf

$M_u = 0.160 klf \cdot (3 ft)^2/8 = 0.18\ k\cdot ft = 2.2\ k\cdot in$

$\phi M_n = 15 k\cdot in$ so $\frac{15-2.2}{15} \cdot 100 = 85\%$ of slab steel is available to help resist flexure.

if (2) 18 ft bays are monolithic, ϕM_n of diaphragm = 8423 k·ft (unreduced)

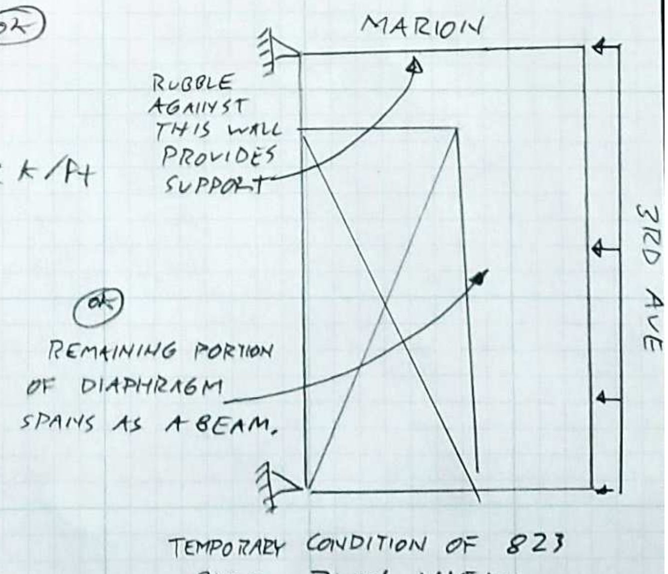
$\phi M_n = 0.85 \cdot 8423 = 7159\ k\cdot ft$ OK

$\phi V_c = 0.6 \cdot 2\sqrt{3000} \cdot 4.5 \cdot 12 = 3.5\ k/ft$

$\phi V_s = 0.6 \cdot (60 \cdot 0.85) \cdot (0.31 in^2 \cdot 0.86) = 8.2\ k/ft$

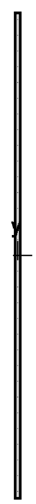
$\phi V_n = 3.5 + 8.2 = 11.7\ k/ft$

ϕV_n total $= 2 \cdot 18 ft \cdot 11.7 k/ft = 421\ k$ OK



TEMPORARY CONDITION OF 823 BLDG - PLAN VIEW

spColumn v6.00
Computer program for the Strength Design of Reinforced Concrete Sections



1. General Information

File Name	D:\Projects\3rd and Columbia De...\Floor Slab.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	3	ksi
E_c	3122.02	ksi
f_c	2.55	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{yt}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	4.5	in
Depth	432	in
A_g	1944	in^2
I_x	3.02331e+007	in^4
I_y	3280.5	in^4
r_x	124.708	in
r_y	1.29904	in
X_o	0	in
Y_o	0	in

3.2. Section Figure

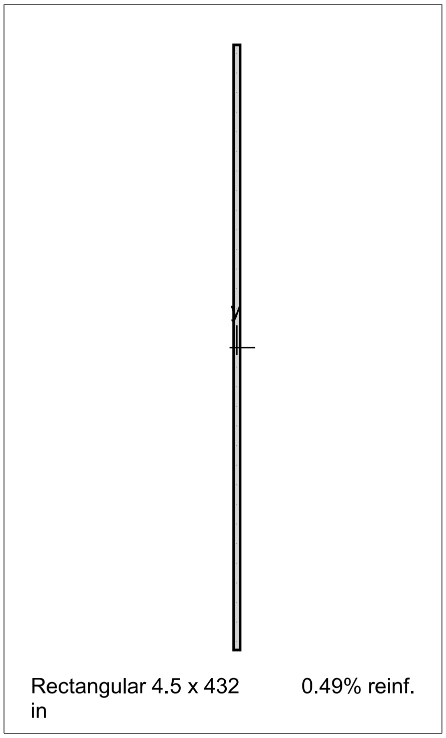


Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in^2		in	in^2		in	in^2
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Total steel area, A_s	9.61 in^2
Rho	0.49 %
Minimum clear spacing	13.37 in

(Note: Rho < 0.50%)

4.4. Bars Provided

Area	X	Y	Area	X	Y	Area	X	Y
in^2	in	in	in^2	in	in	in^2	in	in
0.31	0.0	98.0	0.31	0.0	112.0	0.31	0.0	126.0
0.31	0.0	140.0	0.31	0.0	154.0	0.31	0.0	168.0
0.31	0.0	182.0	0.31	0.0	196.0	0.31	0.0	210.0
0.31	0.0	84.0	0.31	0.0	70.0	0.31	0.0	56.0
0.31	0.0	42.0	0.31	0.0	28.0	0.31	0.0	14.0
0.31	0.0	0.0	0.31	0.0	-14.0	0.31	0.0	-28.0
0.31	0.0	-42.0	0.31	0.0	-56.0	0.31	0.0	-70.0
0.31	0.0	-84.0	0.31	0.0	-98.0	0.31	0.0	-112.0
0.31	0.0	-126.0	0.31	0.0	-140.0	0.31	0.0	-154.0
0.31	0.0	-168.0	0.31	0.0	-182.0	0.31	0.0	-196.0
0.31	0.0	-210.0						

5. Factored Loads and Moments with Corresponding Capacities

No	P_u	M_{ux}	ϕM_{nx}	$\phi M_n/M_u$	NA Depth	d_t Depth	ε_t	ϕ
	kip	k-ft	k-ft		in	in		
1	0.00	4620.00	8423.00	1.823	46.47	426.00	0.02450	0.900